GeoDesign Inc
An NV5 Company

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

1601 E Yesler Way Site
1601 East Yesler Way
Seattle, Washington

For
Shelter Holdings, LLC
September 25, 2020

GeoDesign Project: ShelterH-4-02

GeoDesign Inc
An NV5 Company

September 25, 2020

Shelter Holdings, LLC
11624 SE 5th Street, Suite 210
Bellevue, WA 98005-3590

Attention: Eric Evans

Report of Geotechnical Engineering Services
1601 E Yesler Way Site
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Seattle, Washington
GeoDesign Project: ShelterH-4-02

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the proposed residential development located at 1601 East Yesler Way in Seattle, Washington. Our services for this project were conducted in accordance with our proposal dated October 8, 2019.

We appreciate the opportunity to be of continued service to you. Please contact us if you have questions.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E.
Principal Engineer

cc: Ed Weinstein, Weinstein A+U (via email only)
Renay Luzama, Weinstein A+U (via email only)

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EXECUTIVE SUMMARY

This section provides a summary of the geotechnical considerations associated with the proposed development located at 1601 East Yesler Way in Seattle, Washington. Our conclusions are based on information provided by Shelter Holdings LLC, the design team, and explorations completed at the site. The report describes subsurface conditions and provides geotechnical recommendations in greater detail with respect to the proposed project. The primary geotechnical considerations are summarized as follows:

- The proposed structure can be supported on shallow foundations bearing on the glacially consolidated soil.
- Temporary shoring will be required to construct the proposed basement. Cantilevered shoring may be possible to support the basement excavation adjacent to rights-of-way where some settlement can be tolerated. Anchors may be required where settlement-sensitive utilities or structures are adjacent to the excavation.
- The soil at the bottom of the basement excavation is moisture sensitive and will soften under construction equipment traffic during wet weather. We recommend the contractor place a working blanket to prevent softening of the base of the basement excavation.
- Liquefaction, seismic settlement, lateral spreading, and surface fault rupture are not considered hazards under design levels of ground shaking.
- Cobbles and boulders were encountered in the upper glacially consolidated soil within the excavation zone of the basement. These larger particle sizes can result in more difficult excavations.
- Based on our experience and laboratory testing, the glacially consolidated soil across the site will have low permeability characteristics. If infiltration is considered at the site, small-scale pilot infiltration tests should be completed when the location of proposed infiltration facilities are known. Soil samples for water quality treatment potential can be collected at the time of pilot infiltration tests.

TABLE OF CONTENTS **PAGE NO.**

ACRONYMS AND ABBREVIATIONS

1.0 INTRODUCTION 1
2.0 PROJECT UNDERSTANDING 1
3.0 PURPOSE AND SCOPE 1
4.0 SITE CONDITIONS 2
4.1 Surface Conditions 2
4.2 Subsurface and Groundwater Conditions 2
4.3 Geologic Hazards 3
5.0 SITE DEVELOPMENT RECOMMENDATIONS 3
5.1 Site Preparation 3
5.2 Excavations 4
5.3 Haul Roads and Working Blankets 5
5.4 Shoring 5
5.5 Permanent Slopes 7
5.6 Structural Fill 7
6.0 FOUNDATION SUPPORT 9
6.1 Spread Footings 9
6.2 Slab and Grade 10
7.0 PERMANENT RETAINING STRUCTURES 10
8.0 DRAINAGE CONSIDERATIONS 11
8.1 General 11
8.2 Temporary Drainage 11
8.3 Permanent Surface Drainage 11
9.0 SEISMIC DESIGN CRITERIA 12
9.1 Seismic Design Parameters 12
10.0 OBSERVATION OF CONSTRUCTION 12
11.0 LIMITATIONS 12

REFERENCES 14

FIGURES
Vicinity Map Figure 1
Site Plan Figure 2
Cantilevered and Braced Walls Design Criteria Figure 3
Surcharge-Induced Lateral Earth Pressures Figure 4
Lateral Earth Pressures for Braced Basement Walls Figure 5

TABLE OF CONTENTS **PAGE NO.**

APPENDICES
Appendix A
Field Explorations A-1
Laboratory Testing A-1
Exploration Key Table A-1
Soil Classification System Table A-2
Boring Logs Figures A-1 - A-2
Grain-Size Test Results Figure A-3
Summary of Laboratory Data Figure A-4
Appendix B
Previous Explorations Completed by Others B-1
Site Plan and Exploration Logs

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ASTM	American Society for Testing and Materials
BGS	below ground surface
g	gravitational acceleration (32.2 feet/second^2)
GIS	geographic information system
GPS	global positioning system
H:V	horizontal to vertical
ksf	kips per square foot
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
psf	pounds per square foot
SPT	standard penetration test
µm	micrometer
USGS	U.S. Geological Survey
UST	underground storage tank

1.0 INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation for the proposed development located at 1601 East Yesler Way in Seattle, Washington. The site is shown relative to surrounding features on Figure 1. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

It is our understanding that the 1.4-acre site includes Parcel Number 9826700565 of King County Tax Map NW 4-24-04. The site is currently occupied by a three-story commercial building and associated parking and landscaped areas.

Development will include demolishing the existing structure and constructing a new six-story structure with one level of below-grade parking. We have assumed column loads will range from 400 to 600 kips and floor loads will not exceed 150 psf. The current available schematic plans indicated site grades vary between 242 and 270 within the proposed building area. Preliminary plans indicate a proposed basement finish floor elevation of 241 feet.

3.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed development. The specific scope of our services is summarized as follows:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Coordinated and managed the field explorations, including public and private utility locates and scheduling subcontractors.
- Completed a subsurface exploration program that consisted of drilling two borings to depths between 50.3 and 51.5 feet BGS.
- Installed a vibrating wire piezometer in one of the borings to measure groundwater levels.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Performed a laboratory testing program. The following laboratory tests were completed:
  - Ten moisture content determinations in general accordance with ASTM D2216
  - Five grain-size analyses in general accordance with ASTM C117 or ASTM C136
  - One fines content analysis in general accordance with ASTM D1140
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations, including the following:
  - Recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, and subgrade preparation
  - Recommendations for wet weather construction
  - Soil and groundwater conditions at the site
  - Discussion of seismic hazards at the site, including liquefaction and lateral spreading
  - Recommendations for temporary shoring

- Recommendations for foundation support for the proposed building
- Recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures
- General recommendations for subsurface drainage
- Seismic design parameters in accordance with the 2015 IBC

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The approximately 1.4-acre site is bound by East Yesler Way to the north, 17th Avenue South to the east, South Washington Street to the south, and 16th Avenue South to the west. The site is currently occupied by an approximately 70,440-square-foot, L-shaped, commercial structure and associated paved parking and landscaped areas. The site slopes down to the southwest with approximately 28 feet of elevation change. Rockeries up to approximately 4 feet tall are present along the southwestern end of the property.

4.2 SUBSURFACE AND GROUNDWATER CONDITIONS

4.2.1 General

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths between 51.3 and 51.5 feet BGS. The approximate exploration locations are shown on Figure 2. Descriptions of our field exploration and laboratory testing programs, logs of the explorations, and the results of laboratory testing are presented in Appendix A.

In addition, we reviewed subsurface information from a previous study at the site in 1984 by Shannon & Wilson, Inc. to support the existing site development. The site plan and exploration logs from this study are presented in Appendix B. The test pits showed variable fill to depths of up 3 feet BGS overlying glacial till to the maximum depth explored of 9 feet BGS. Some of the fill was likely removed during construction of the current development.

Our borings were completed in the existing AC in the parking lot and trash enclosure area. Subsurface conditions below the pavement section generally consist of glacially consolidated soil. The following sections provide a more detailed description of the units encountered during our explorations.

4.2.2 AC Pavement

We observed the AC pavement is generally 2 to 3 inches thick and underlain by approximately 4 inches of aggregate base

4.2.3 Glacially Consolidated Soil

Glacial till consisting of very dense, silty sand with gravel and variable amounts of cobbles and boulders is present beneath the pavement section to depths of approximately 18 and 38 feet BGS in borings B-1 and B-2, respectively.

Advance outwash consisting of very dense, silty sand and sand with silt is present below the glacial till the maximum depth explored of 51.5 feet BGS. Trace to minor amounts of gravel

were observed in portions of the advance outwash. The moisture content of the glacially consolidated soil ranged from 7 and 20 percent at the time of our explorations.

4.2.4 Groundwater

Groundwater was observed during drilling at depths of approximately 37 and 34 feet BGS in borings B-1 and B-2, respectively. To measure groundwater levels after drilling, we installed a vibrating wire piezometer in boring B-1 at a depth of 44.4 feet BGS. Table 1 provides the groundwater measurements obtained from the piezometer after drilling.

Table 1. Groundwater Measurements

Date	Depth (feet BGS)	Elevation[1] (feet)
10/29/19	38.0	220.0
11/1/19	38.1	219.9
11/7/19	38.4	219.6

1. Elevation of boring B-1 monument assumed to be 258 feet based on City of Seattle GIS Maps.

We anticipate that groundwater levels may fluctuate during the year with response to the seasons. We anticipate that perched water may be present during periods of wet weather within sandier zones in the glacially deposited soil.

4.3 GEOLOGIC HAZARDS

4.3.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking. Soil that is susceptible to liquefaction was not encountered in our explorations.

4.3.2 Fault Surface Rupture

We reviewed USGS seismic hazard maps and published geologic maps for the area. The nearest fault is the Seattle fault zone mapped approximately 1.6 miles to the south of the site. Therefore, it is our opinion that the potential for surface fault rupture at the site is low.

5.0 SITE DEVELOPMENT RECOMMENDATIONS

5.1 SITE PREPARATION

5.1.1 Stripping and Grubbing

Stripping and grubbing will be required to remove trees and shrubs in the landscaped areas. The existing root zone material should be removed from all proposed structure and pavement areas. The actual stripping and grubbing depth should be based on field observations at the time of

construction. Stripping and grubbing should extend at least 5 feet beyond the limits of proposed structural areas. Organic material should be transported off site for disposal or used as fill in landscaped areas.

5.1.2 Demolition

Demolition should include the complete removal of the existing building, retaining walls, rockeries, concrete pavement, sidewalks, utilities, USTs, and any other underlying structural elements. The slab, walls, and footings of the existing building should be completely removed. Any monitoring wells or USTs should be abandoned in accordance with state and local regulations prior to site development. Abandoned utility lines under new structural components should be completely removed. Excavations resulting from demolition of existing improvements should be backfilled with compacted structural fill as recommended in this report. The base of the excavations should be excavated to expose firm subgrade. The sides of the temporary excavations should be cut into firm material and sloped no steeper than 1½H:1V.

5.1.3 Subgrade Evaluation

A member of our geotechnical staff should observe the exposed footing, slab, and pavement subgrade after stripping, excavation, and placement of structural fill have been completed to confirm that there are no areas of unsuitable or unstable soil. The subgrade should be evaluated using moisture-density testing, a hand probe, and/or proof rolling with a fully loaded dump truck (or similar heavy, rubber tire construction equipment). If soft, loose, or otherwise unsuitable soil is found at the subgrade level, we recommend that the soil be over-excavated and replaced with structural fill.

5.2 EXCAVATIONS

Excavations will be required for the installation of the underground parking level, new footings, utilities, and other earthwork. Cobbles and boulders were encountered in the glacial till soil and can lead to difficult excavation conditions.

Excavations in the silty sand may be prone to raveling. Excavations deeper than 4 feet BGS will require shoring or should be sloped. Sloped excavations may be used to vertical depths of 10 feet BGS and should have side slopes no steeper than 1½H:1V, provided groundwater seepage does not occur. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. The contractor should be responsible for selecting the appropriate shoring system.

Excavations should not be allowed to undermine adjacent improvements. If existing roads or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet from the base of the existing footings. Excavations that must be inside of this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

We anticipate that excavations for this project will not extend below the groundwater level. We anticipate that significant dewatering will not be required for this project. Perched water or rainwater can likely be removed by pumping from sumps located within the excavation.

While we have described certain approaches to performing excavations, it is the contractor's responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All excavations should be in accordance with applicable OSHA and state regulations.

5.3 HAUL ROADS AND WORKING BLANKETS

The silty sand soil at the site is easily disturbed during the wet season. Granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the subgrade soil is more than a few percentage points above optimum. If not carefully executed, site preparation and excavation activities can create extensive soft areas, resulting in significant repair costs. Earthwork should be planned and executed to minimize subgrade disturbance. Haul roads subject to repeated construction traffic will require a minimum of 18 inches of imported granular material. For light staging areas near existing site grades, 12 inches of imported granular material will likely be sufficient.

Based on the soil conditions at the site, we recommend that a working blanket consisting of at least 12 inches of granular material be constructed to support construction traffic operating at the base of the mass excavation. The subgrade at the base of the excavation should be evaluated to determine suitable conditions prior to constructing the working blanket.

These are guidelines; the contractor should select the thickness of haul roads and working blankets based on the anticipated construction traffic volumes and loads. The imported granular material for haul roads and staging areas should consist of crushed rock that is well graded and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve.

5.4 SHORING

5.4.1 General

Temporary shoring will be required to construct the proposed basement associated with the project. Based on our experience, conventional soldier pile and timber lagging are the most appropriate forms of temporary shoring. If the basement cuts are more than 15 feet, movement-sensitive structures are located within a horizontal distance equal to the wall height, or additional basement levels are added, tiebacks will be required to resist lateral loads.

Conventional soldier pile shoring and wood lagging can be designed using the values presented on Figure 3. At-rest pressures should be used in design where shoring is directly adjacent to existing structures. These values do not include surcharged-induced earth pressures. Figure 4 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil to account for traffic loads where appropriate. If the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

Table 2. Lateral Earth Pressure Increase Factors for Sloped Soil

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27
35	3.35

5.4.2 Soldier Piles

We recommend that the embedded portion of the solider piles be at least 2 feet in diameter and extend a minimum of 10 feet below the base of the excavation. If tiebacks are required, the axial component of the load can be resisted through end bearing and skin friction along the solider pile. An allowable end bearing pressure of 20 ksf can be used for soldier piles supported on undisturbed glacial soil. If necessary, a skin friction of 1.5 ksf may be used for the embedded portion of the pile to resist axial forces.

5.4.3 Tieback Shoring

Soldier pile and tieback walls should be designed as shown on Figure 3. The bonded zone for the tieback anchors should be maintained outside of the "no load zone" show on Figure 3. We anticipate that the tieback anchors can achieve an allowable bond strength of at least 4 kips per foot in glacially consolidated soil, depending on the method of anchor construction. A variety of methods are available for construction of tieback anchors and higher adhesion values may be possible. Therefore, we recommend that the contractor be responsible for selecting the appropriate bond strength, bonded length, and installation methods to achieve the required anchor capacity.

We recommend that performance testing be conducted on a minimum of two anchors in each soil type. Performance testing should be completed to 200 percent of the design load. The purpose of this testing is to verify the installation procedure selected by the contractor before a large number of anchors are installed. We recommend that proof testing to 133 percent of the design load be conducted on all other production anchors. Performance and proof testing should be performed in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

5.4.4 Lagging

Lagging should consist of cross members between vertical supports capable of resisting horizontal earth pressures equal to one-half of the earth pressures used to design the shoring. This one-half reduction is an approximation of the preferential redistribution of earth pressures on the stiff, tied-back soldier piles compared to the relatively flexible lagging between the piles. Soldier pile spacing greater than four times the pile diameter will decrease the effect of arching

between piles, resulting in higher lagging earth pressures. A more accurate earth pressure distribution for the lagging can be provided if the pile spacing is greater than four times the pile diameter.

Lagging pressures can increase temporarily when the anchors are tested, particularly on the lagging located near the location of an anchor. This effect on lagging can be reduced if walers or a testing beam are used to span to adjacent soldier piles.

Lagging will likely consist of wood or shotcrete. We recommend prompt and careful installation of lagging to maintain the integrity of the excavation, particularly in areas of seepage and raveling soil. All voids behind the lagging should be fully backfilled with grout. Where shoring supports surcharge loads from adjacent structures, we recommend no more than 4 feet of unsupported exposed face during construction of the shoring.

5.5 PERMANENT SLOPES

Permanent cut or fill slopes should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Upslope buildings, access roads, and pavement should be set back a minimum of 5 feet from the crest of such slopes. Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.6 STRUCTURAL FILL

Structural fill includes fill beneath foundations, slabs, pavement, any other areas intended to support structures, or within the influence zones of structures. Structural fill should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 3 inches in diameter. Recommendations for suitable fill material are provided in the following sections.

5.6.1 On-Site Soil

The on-site soil will be suitable for use as structural fill only if it can be moisture properly moisture conditioned. Based on our experience, the on-site silty soil is sensitive to small changes in moisture content and may be difficult, if not impossible, to compact adequately during wet weather or when its moisture content is more than a few percentage points above optimum. Laboratory testing indicates that the moisture content of the native soil is variable and may require drying if it is used as structural fill. We recommend using imported granular material for structural fill if the moisture content of the on-site soil cannot be reduced.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 10 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

5.6.2 Imported Granular Material

Imported granular material should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The percentage of fines can be increased to

12 percent if the fill is placed during dry weather and provided the fill material is moisture conditioned, as necessary, for proper compaction. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should have a maximum thickness of 15 inches and should be compacted by rolling with a smooth-drum, non-vibratory roller.

5.6.3 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within pavement areas and building pads, the upper portion of the trench backfill should consist of well-graded granular material, with a maximum particle size of 2½ inches. Trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

5.6.4 Stabilization Material

Material used to stabilize staging areas, haul roads, and utility trench subgrade should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand with a maximum particle size of 4 inches; should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve; and should have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. The stabilization material should be placed in one lift and compacted to a well-keyed, firm condition.

5.6.5 Drain Rock

Drain rock should consist of angular, granular, open-graded material with a maximum particle size of 2 inches. The material should be free of roots, organic matter, and other unsuitable materials; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

5.5.6 Floor Slab Base Rock

Imported granular material placed beneath building floor slabs should be clean crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular material should have a maximum particle size of 1½ inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

5.6.7 Recycled Concrete

Recycled concrete can be used for structural fill provided the concrete is broken to a maximum particle size of 3 inches. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.0 FOUNDATION SUPPORT

6.1 SPREAD FOOTINGS

6.1.1 General

Based on the proposed basement depth and the results of our soil explorations, it is our opinion that conventional spread footings resting on glacially consolidated soil are suitable to support the anticipated building loads. Due to the depth of the basement, fills are not anticipated at the footing elevations; however, all fill should be completely removed beneath footings to glacially consolidated soil and replaced with structural fill if encountered.

6.1.2 Dimensions and Capacities

Footings bearing directly on the glacially consolidated soil can be proportioned for a maximum allowable soil bearing pressure of 8,000 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and may be doubled when considering seismic or wind loads. The weight of the footing and any overlying backfill can be ignored in calculating footing loads.

All isolated and linear wall footings should have a minimum horizontal dimension of 2 feet and 1.5 feet, respectively. The bearing surfaces of all footings should be embedded at least 1.5 feet below the lowest adjacent grade.

Shallow foundations designed according to our recommendations will experience total post-construction settlement of less than 1 inch. Differential settlement will be less than ½ inch between adjacent footings. We expect that most settlement will occur during construction as loads are applied.

6.1.3 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction equal to 0.40 can be used for resistance to sliding for footings in contact with crushed rock pads; this should be reduced to 0.30 for footings on native soil.

6.1.4 Construction Considerations

All footing subgrades should be evaluated by a representative of GeoDesign to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organics,

unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

If foundation construction is planned during wet weather, we recommend that all footing subgrades be protected using lean concrete mud mats. The mud mat should be placed the same day that the footing subgrade is evaluated.

6.2 SLAB ON GRADE

Satisfactory subgrade support for slabs supporting floor loads of up to 100 psf can be obtained on the native soil or on structural fill. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The imported granular material should have a maximum particle size of 1½ inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

A soil subgrade modulus of 150 pci should be used to design floor slabs supported on near-surface soil or structural fill. Settlement of the slab supporting the anticipated design loads and constructed as recommended is not expected to exceed approximately 1 inch of total and differential settlement.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

7.0 PERMANENT RETAINING STRUCTURES

Embedded walls that are braced should be designed using the earth pressure diagram provided on Figure 5. Walls should be designed to resist the earth pressures developed by the shoring system, unless the shoring is designed as a permanent installation. Permanent retaining structures not in contact with temporary shoring should be designed as recommended below.

Cantilevered walls not restrained from rotation should be designed using an equivalent fluid pressure of 35 pcf. An equivalent fluid pressure of 55 pcf should be used for retaining walls restrained from rotation. These values do not consider hydrostatic pressures.

Seismic earth pressures on retaining walls should be designed using a value of 7H pounds per linear foot of wall, where H is the wall height. This seismic pressure should be applied as a uniform rectangular pressure throughout the excavated depth of the retaining wall.

The permanent retaining walls should be fully drained if they are not designed to resist hydrostatic pressures. This may be accomplished through the use of prefabricated

geocomposite drainage panels placed against the face of the temporary wall. The panels should extend into a conventional horizontal foundation drain at the base of the wall. The foundation drain shall consist of at least 18 inches of drain rock. The drain rock should extend at least 12 inches below the base of the adjacent basement slab. The drain rock should be wrapped in a non-woven geotextile to prevent the migration of fines into the rock. A perforated drainage pipe shall be placed within the drain rock a maximum of 4 inches above the base of the trench. The perforated drainage pipe should carry collected water to a sump and pump for disposal. Moisture-proofing along the back of the permanent wall should be specified by the project architect.

Backfill material placed behind walls should extend a horizontal distance of ½H, where H is the height of the retaining wall. Backfill should consist of well-graded sand or gravel with not more than 5 percent by dry weight passing the U.S. Standard No. 200 sieve.

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after construction, unless survey data indicates that settlement is complete prior to that time.

8.0 DRAINAGE CONSIDERATIONS

8.1 GENERAL

We recommend that roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to the building be sloped to facilitate positive drainage away from the building.

8.2 TEMPORARY DRAINAGE

During grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

8.3 PERMANENT SURFACE DRAINAGE

The finished ground surface around the building should be sloped away from the foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers

should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to the building without providing means for positive drainage (i.e., swales or catch basins).

9.0 SEISMIC DESIGN CRITERIA

9.1 SEISMIC DESIGN PARAMETERS

We understand that the seismic design criteria for construction of the new building will be based on the 2015 IBC. Based on the results of our explorations, a seismic Site Class C is the most appropriate for the site. Table 3 presents the site design parameters for the site.

Table 3. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
Spectral Acceleration, S (MCE)	S_s = 1.366 g	S_1 = 0.528 g
Site Class	C	
Site Coefficient, F	F_a = 1.000	F_v = 1.300
Spectral Acceleration Parameters, S_M (MCE)	S_{MS} = 1.366 g	S_{M1} = 0.687 g
Design Spectral Acceleration Parameters, S_D	S_{DS} = 0.911 g	S_{D1} = 0.458 g

10.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

11.0 LIMITATIONS

We have prepared this report for use by Shelter Holdings, LLC and members of the design and construction team for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

♦ ♦ ♦

We appreciate the opportunity to be of service to you. Please call if you have questions.

Sincerely,

GeoDesign, Inc.

Joe Westergreen, P.E.
Project Engineer

Brett A. Shipton, P.E.
Principal Engineer

Signed 09/25/2020

REFERENCES

IBC 2015. International Building Code, 2012 Edition.

Post-Tensioning Institute, 2014. *Recommendations for Prestressed Rock and Soil Anchors.*

FIGURES

GeoDesign Inc AN NV5 COMPANY	SHELTERH-4-02	**VICINITY MAP**	
	SEPTEMBER 2020	1601 E YESLER WAY SITE SEATTLE, WA	**FIGURE 1**

Printed By: aday | Print Date: 9/25/2020 9:48:36 AM
File Name: J:\S-Z\ShelterH\ShelterH-4\ShelterH-4-02\Figures\CAD\ShelterH-4-02-VM01.dwg | Layout: FIGURE 1

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File Name: J:\S-Z\ShelterH\ShelterH-4\ShelterH-4-02\Figures\CAD\ShelterH-4-02-SP01.dwg | Layout: FIGURE 2

LEGEND:
B-1 BORING

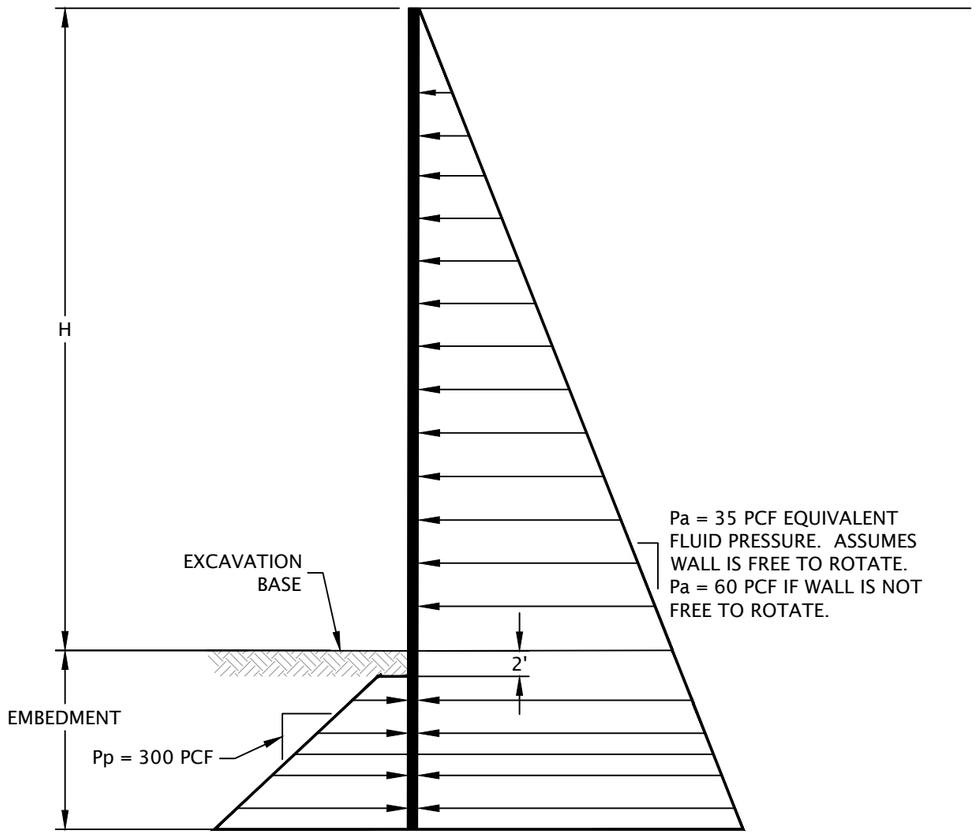
0
40
80
(SCALE IN FEET)

N

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, OCOTBER 22, 2019

GeoDesign Inc an NV5 company	SHELTERH-4-02	SITE PLAN
	SEPTEMBER 2020	1601 E YESLER WAY SITE SEATTLE, WA
		FIGURE 2

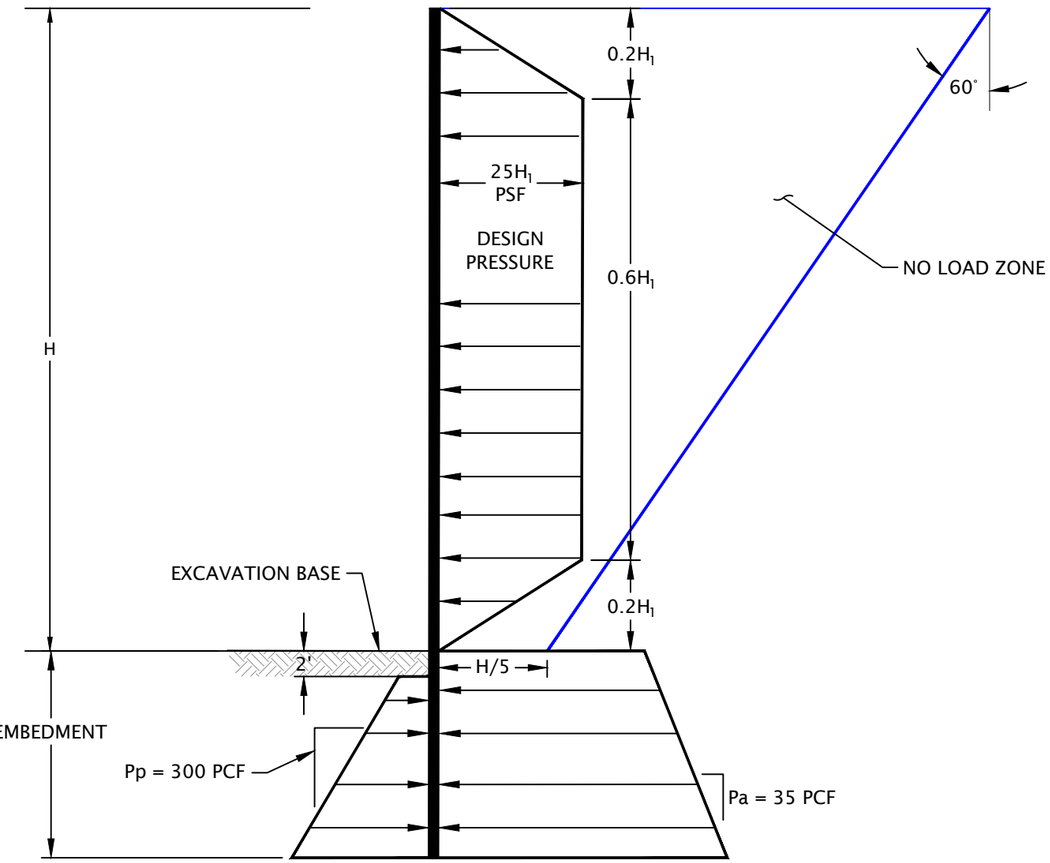
RECOMMENDED DESIGN PARAMETERS FOR CANTILEVERED WALL


H
EXCAVATION BASE
Pa = 35 PCF EQUIVALENT FLUID PRESSURE. ASSUMES WALL IS FREE TO ROTATE. Pa = 60 PCF IF WALL IS NOT FREE TO ROTATE.
2'
EMBEDMENT
Pp = 300 PCF

EXPLANATION:
H = HEIGHT OF EXPOSED SOLDIER PILE (IN FEET)

NOTES:
1. FIGURE DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. PASSIVE EARTH PRESSURES INCLUDE A FACTOR OF SAFETY OF 1.5.
3. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
4. PASSIVE PRESSURE ACTS OVER 2.5X THE PILE WIDTH.
5. ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION.

RECOMMENDED DESIGN PARAMETERS FOR BRACED WALLS



EXPLANATION:
H = HEIGHT OF EXPOSED SOLDIER PILE (IN FEET)

NOTES:
1. FIGURE DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. PASSIVE EARTH PRESSURES INCLUDE A FACTOR OF SAFETY OF 1.5.
3. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
4. PASSIVE PRESSURE ACTS OVER 2.5X THE PILE WIDTH.
5. ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION.
6. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF THE DESIGN LOAD.

CANTILEVERED AND BRACED WALLS DESIGN CRITERIA

FIGURE 3

1601 E YESLER WAY SITE
SEATTLE, WA

SHELTERH-4-02

SEPTEMBER 2020

GEODESIGN INC AN NV5 COMPANY

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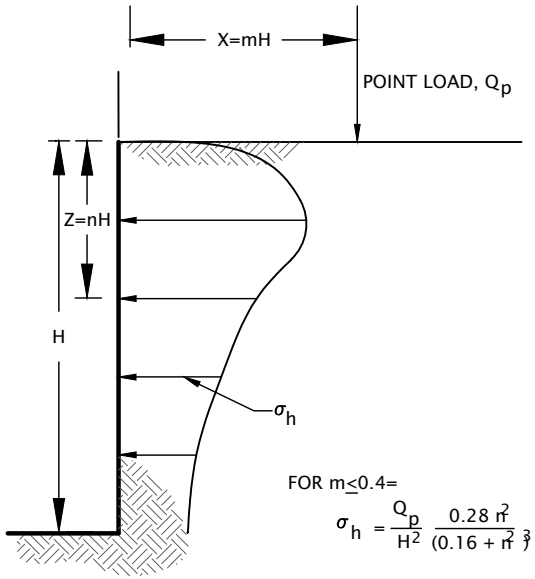
Printed By: aday | Print Date: 9/25/2020 9:48:49 AM
File Name: J:\S-Z\ShelterH\ShelterH-4\ShelterH-4-02\Figures\CAD\ShelterH-4-02-DET02.dwg | Layout: FIGURE 4

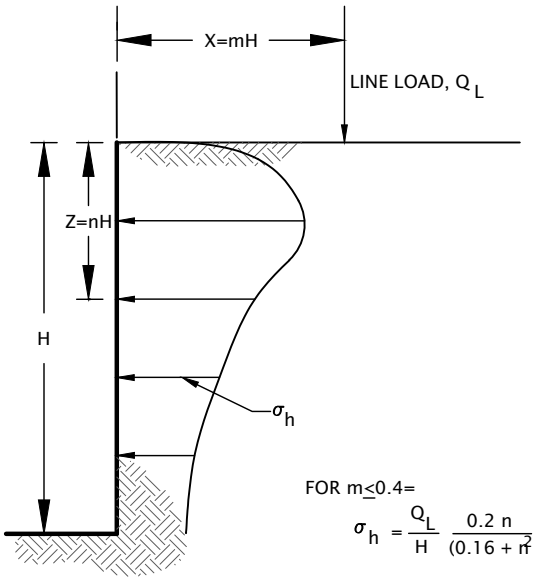


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$



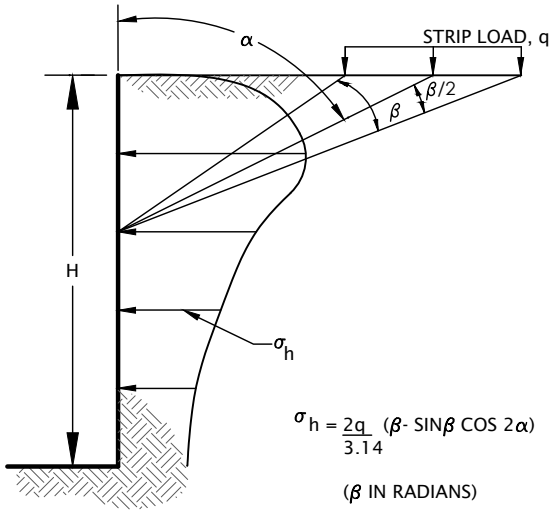
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{0.2\, n}{(0.16 + n^2)^2}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

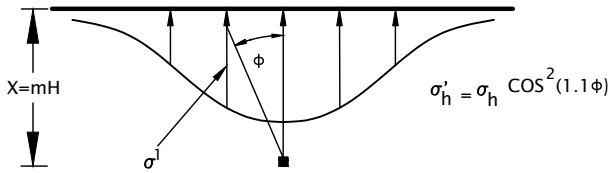
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14} (\beta - \sin\beta \cos 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL



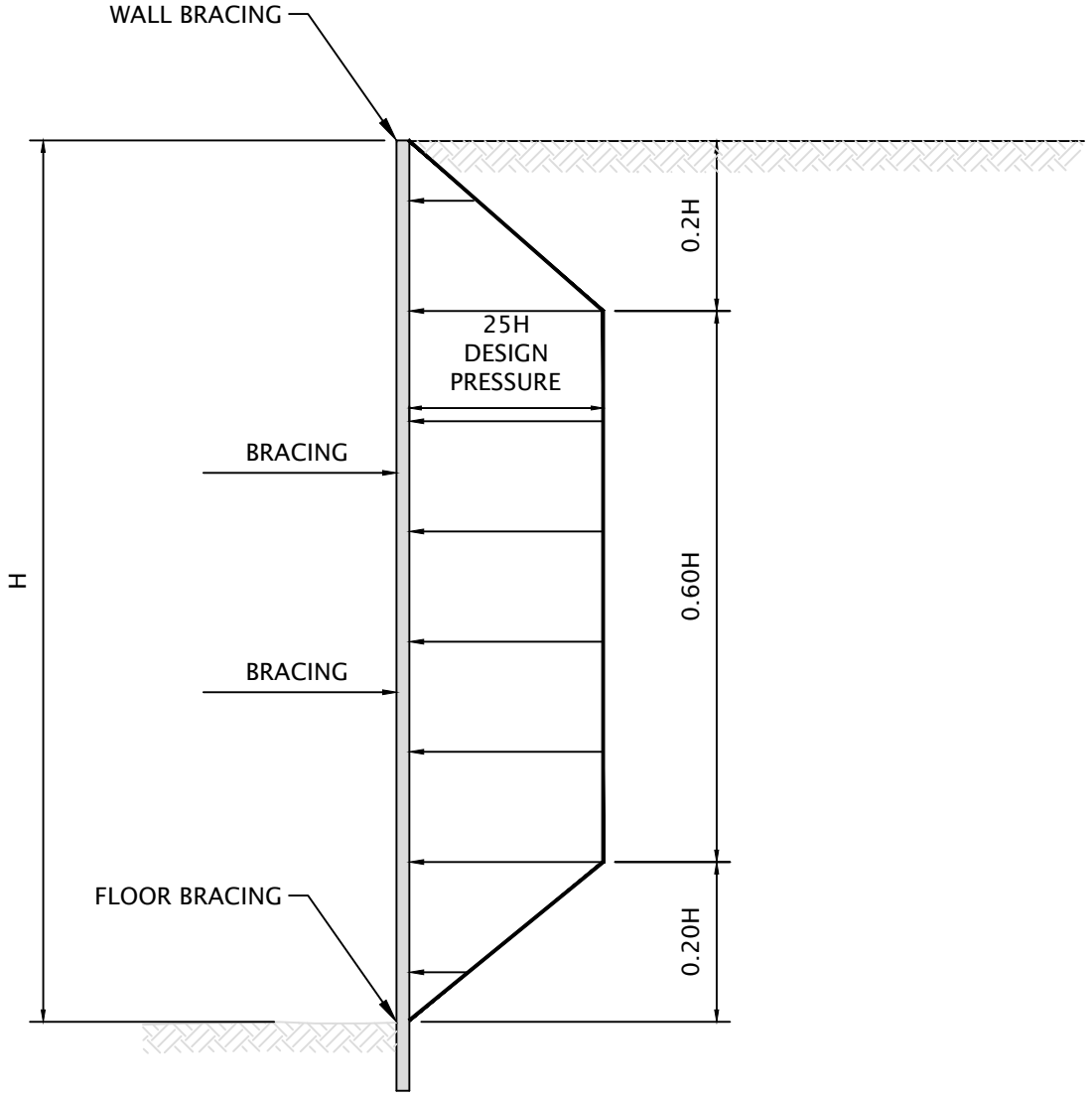
DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES IN THIS FIGURE ARE UNFACTORED.

GEODESIGN INC AN NV5 COMPANY	SHELTERH-4-02	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**	
	SEPTEMBER 2020	1601 E YESLER WAY SITE SEATTLE, WA	**FIGURE 4**



NOT TO SCALE

EXPLANATION:

H DEPTH OF EXCAVATION/BASEMENT WALL IN FEET

NOTES:

1. SURCHARGE LOADS ASSOCIATED WITH CONSTRUCTION ACTIVITIES AND TRAFFIC SHOULD BE ADDED TO THE EARTH PRESSURE SHOWN ABOVE.
2. LATERAL EARTH PRESSURES ARE UNFACTORED.
3. ASSUMES DRAINAGE IS INSTALLED BEHIND WALLS TO PREVENT BUILDUP OF HYDROSTATIC PRESSURE.

GEODESIGN INC AN NV5 COMPANY	SHELTERH-4-02	**LATERAL EARTH PRESSURES FOR BRACED BASEMENT WALLS**
	SEPTEMBER 2020	1601 E YESLER WAY SITE SEATTLE, WA — **FIGURE 5**

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface conditions at the site were explored by drilling two borings (B-1 and B-2) to depths of up to 51.5 feet BGS. The borings were completed on October 17, 2019 by BoreTec1, Inc. of Valleyford, Washington, using an excavator-mounted drill rig and hollow-stem auger drilling techniques. The exploration logs are presented in this appendix. The locations of the explorations were determined in the field by using hand-held GPS equipment. This information should be considered accurate to the degree implied by the methods used.

SOIL SAMPLING

We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Samples were collected from the borings using a 1½-inch-inside diameter, split-spoon sampler (SPT sampler). The split-spoon sampling was conducted in general accordance with ASTM D1586. The 1½-inch-inside diameter, split-spoon samplers were driven into the soil with 140-pound hammer free falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the boring logs, unless otherwise noted. Sampling methods and intervals are shown on the exploration logs.

The SPT blows completed by BoreTec1, Inc. were conducted using two wraps around a cathead.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

GRAIN-SIZE ANALYSIS

We completed grain-size testing on select soil samples in order to determine the distribution of soil particle sizes. The testing was completed in general accordance with ASTM C136 and ASTM C117. The test results are presented in this appendix.

MOISTURE CONTENT

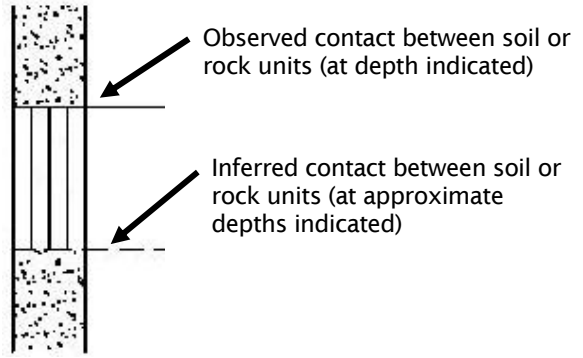
We tested the moisture content of select soil samples in general accordance with ASTM D2216. The moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

FINES CONTENT

We completed fines content testing on a select soil sample in order to determine the soil characteristics. The fines content is the weight of particles smaller than 75 µm expressed as a percentage of the dry weight of the soil sample. The testing was completed in general accordance with ASTM D1140. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types


Observed contact between soil or rock units (at depth indicated)
Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

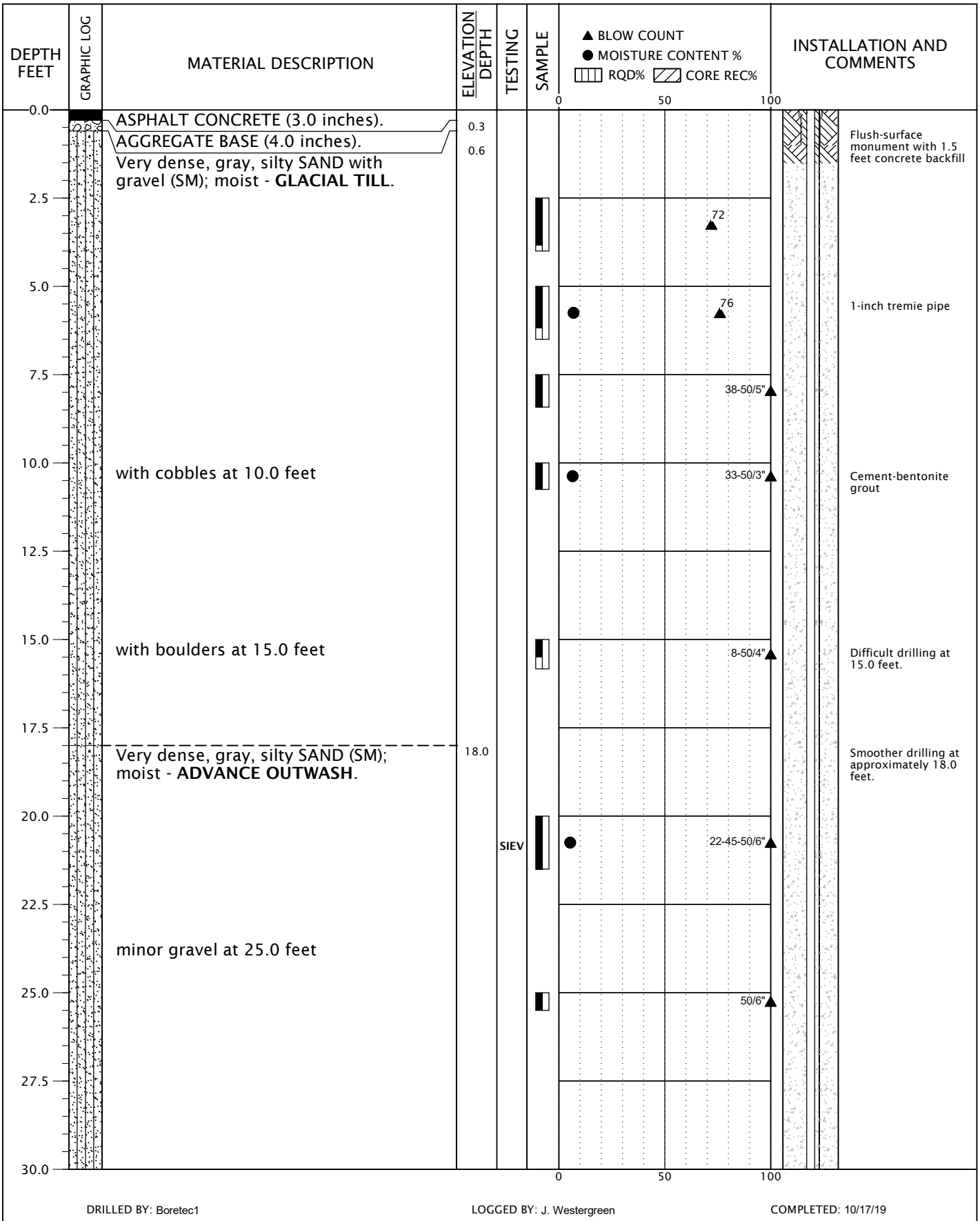
Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

GEODESIGN INC AN NV5 COMPANY

SOIL CLASSIFICATION SYSTEM

TABLE A-2



DRILLED BY: Boretec1 LOGGED BY: J. Westergreen COMPLETED: 10/17/19

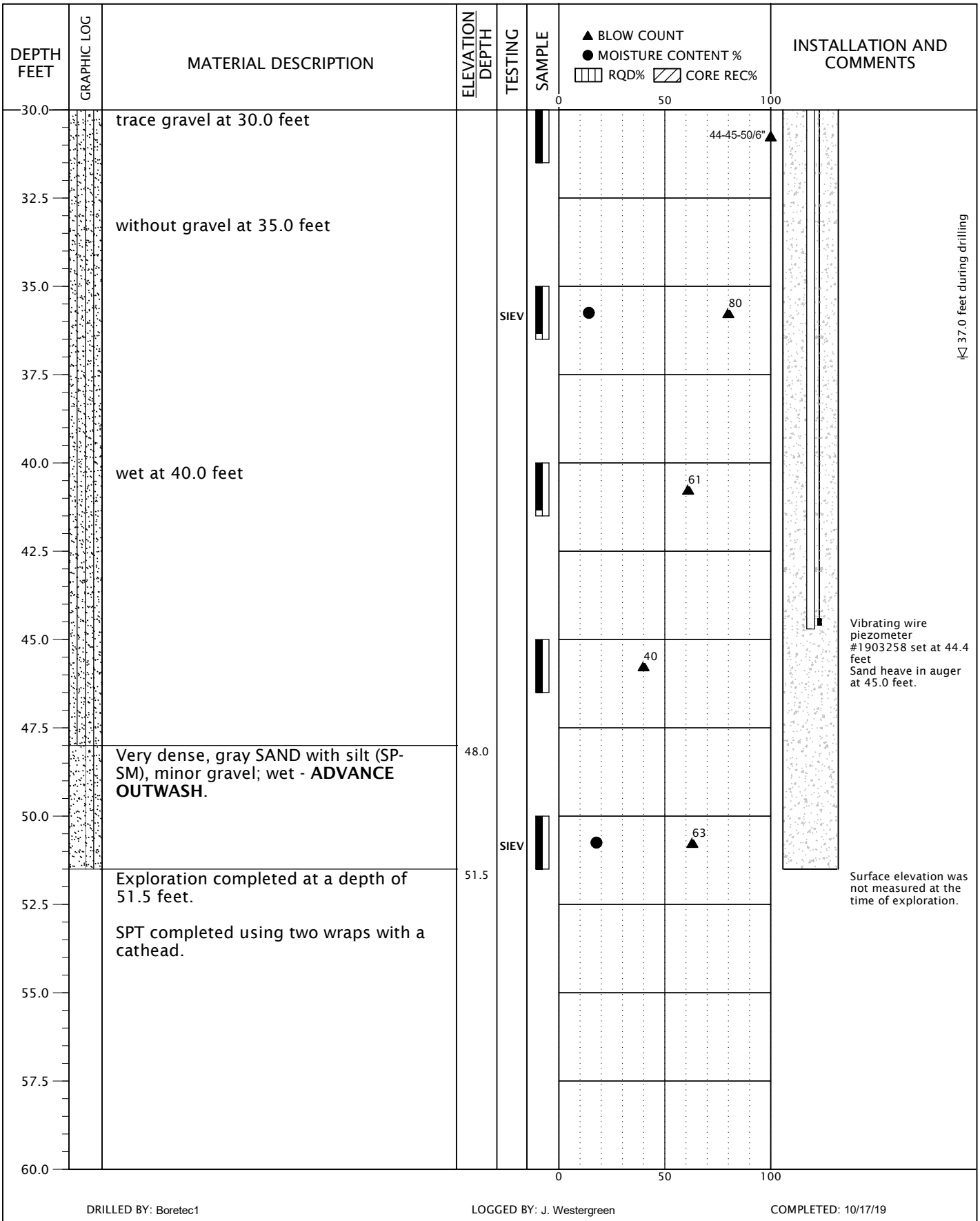
BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 4 1/4 inches

GEODESIGN INC AN NV5 COMPANY

	SHELTERH-4-02	**BORING B-1**
	SEPTEMBER 2020	1601 E YESLER WAY SITE SEATTLE, WA

FIGURE A-1

BORING LOG - GDI-NV5 - 1 PER PAGE SHELTERH-4-02-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 9/25/20:KM



DRILLED BY: Boretec1 LOGGED BY: J. Westergreen COMPLETED: 10/17/19

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 4 1/4 inches

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SHELTERH-4-02

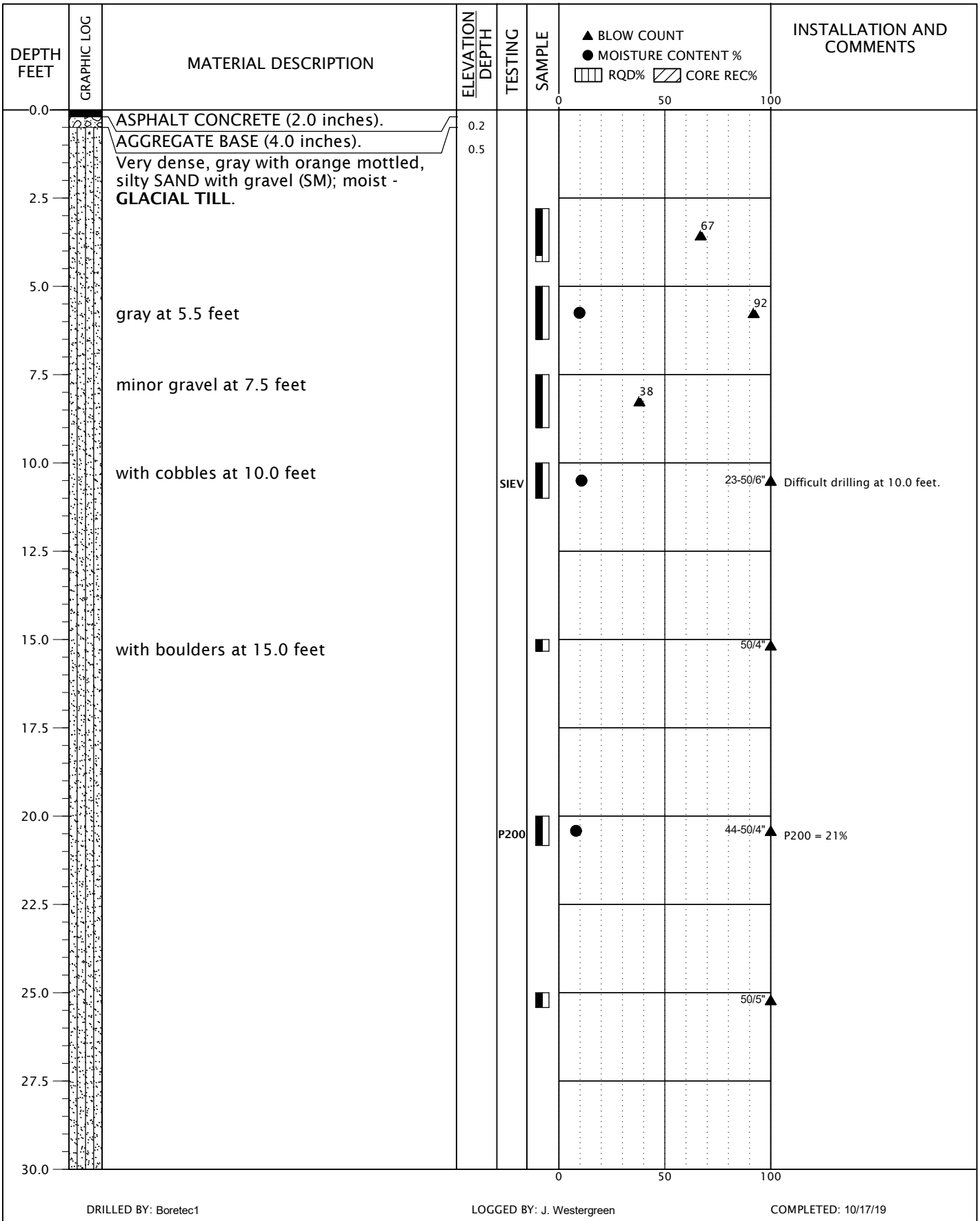
BORING B-1
(continued)

SEPTEMBER 2020

1601 E YESLER WAY SITE
SEATTLE, WA

FIGURE A-1

BORING LOG - GDI-NV5 - 1 PER PAGE SHELTERH-4-02-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 9/25/20:KM



DRILLED BY: Boretec1

LOGGED BY: J. Westergreen

COMPLETED: 10/17/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 4 1/4 inches

GeoDesign Inc
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SHELTERH-4-02

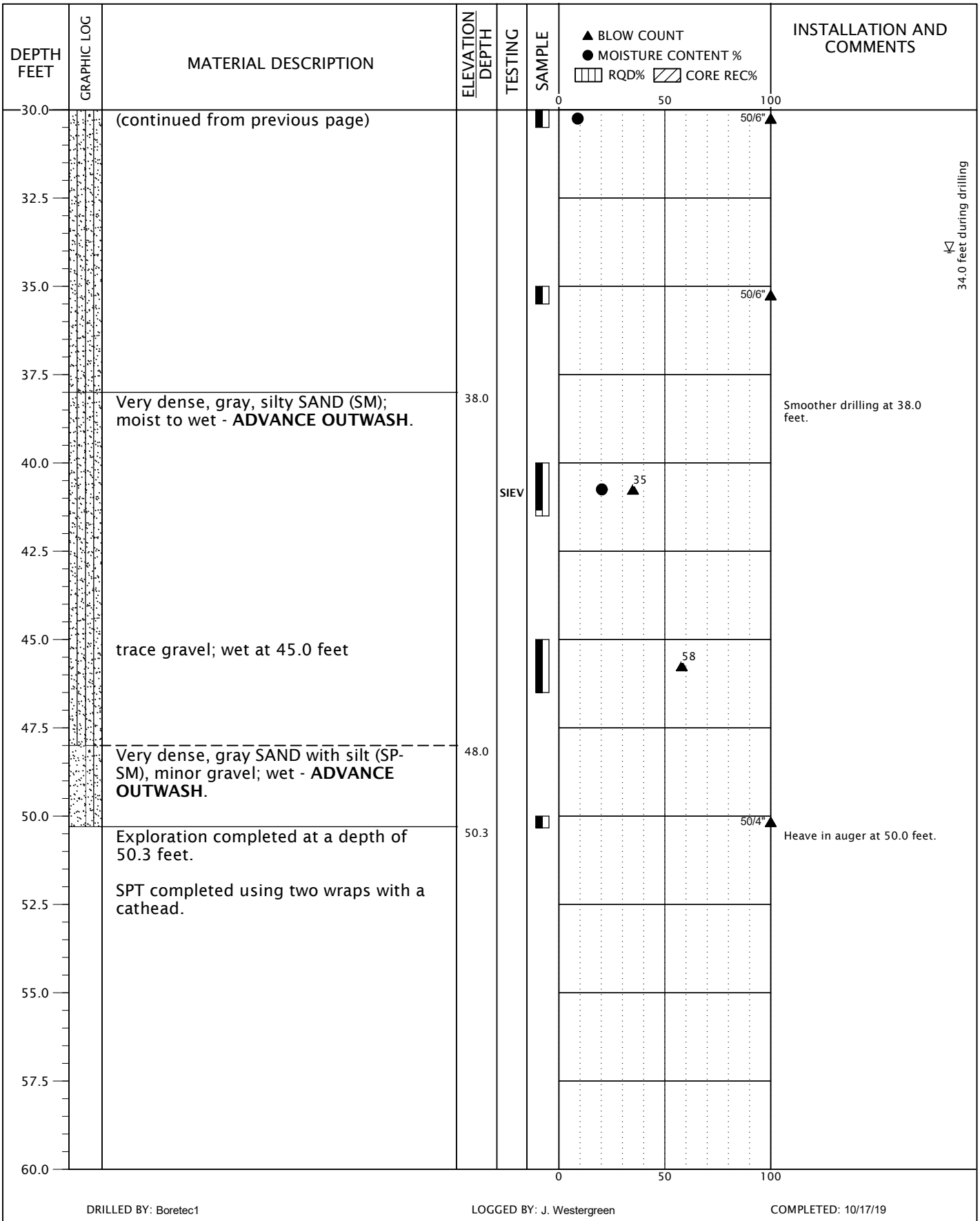
SEPTEMBER 2020

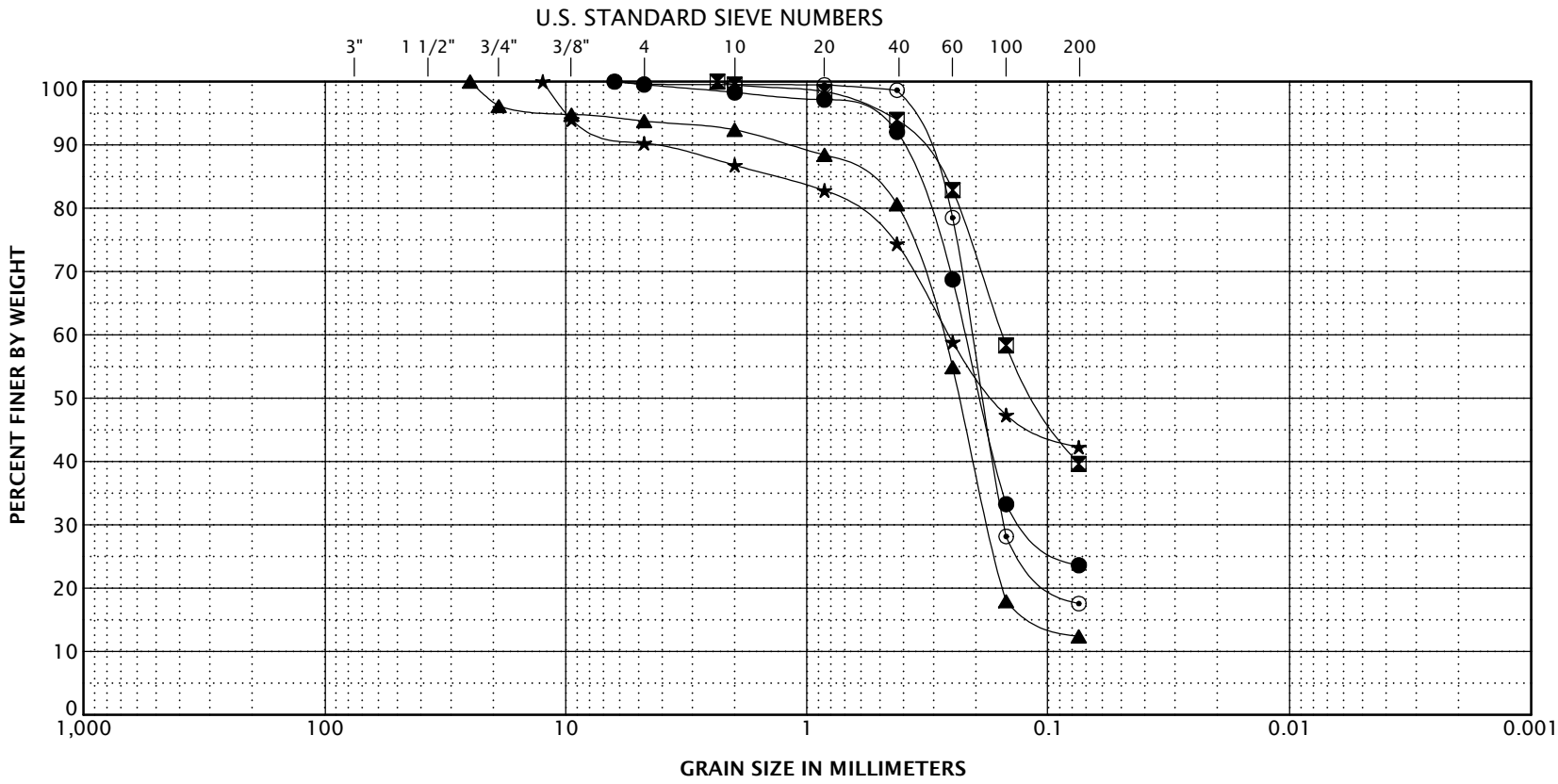
BORING B-2

1601 E YESLER WAY SITE
SEATTLE, WA

FIGURE A-2

BORING LOG - GDI-NV5 - 1 PER PAGE SHELTERH-4-02-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 9/25/20:KM


DEPTH FEET
GRAPHIC LOG
MATERIAL DESCRIPTION
ELEVATION DEPTH
TESTING
SAMPLE
▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%
INSTALLATION AND COMMENTS
0
50
100
30.0
32.5
35.0
37.5
40.0
42.5
45.0
47.5
50.0
52.5
55.0
57.5
60.0
(continued from previous page)
50/6"
50/6"
34.0 feet during drilling
Very dense, gray, silty SAND (SM); moist to wet - ADVANCE OUTWASH.
38.0
Smoother drilling at 38.0 feet.
SIEV
35
trace gravel; wet at 45.0 feet
58
Very dense, gray SAND with silt (SP-SM), minor gravel; wet - ADVANCE OUTWASH.
48.0
50/4"
Heave in auger at 50.0 feet.
Exploration completed at a depth of 50.3 feet.
50.3
SPT completed using two wraps with a cathead.
DRILLED BY: Boretec1
LOGGED BY: J. Westergreen
COMPLETED: 10/17/19
BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 4 1/4 inches
GEODESIGN INC
AN NV5 COMPANY
SHELTERH-4-02
BORING B-2
(continued)
SEPTEMBER 2020
1601 E YESLER WAY SITE
SEATTLE, WA
FIGURE A-2
BORING LOG - GDI-NV5 - 1 PER PAGE SHELTERH-4-02-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 9/25/20:KM



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	B-1	20.0	5	0.22	0.19	0.12			0	76	24	
⊠	B-1	35.0	14	0.16	0.11				0	60	40	
▲	B-1	50.0	18	0.28	0.23	0.18			6	81	12	
★	B-2	10.0	11	0.26	0.17				10	48	42	
⊙	B-2	40.0	20	0.21	0.19	0.15			0	82	18	

GeoDesign Inc
An NV5 Company

SHELTERH-4-02

GRAIN-SIZE TEST RESULTS

SEPTEMBER 2020

1601 E YESLER WAY SITE
SEATTLE, WA

FIGURE A-3

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	5.0		7							
B-1	10.0		6							
B-1	20.0		5		0	76	24			
B-1	35.0		14		0	60	40			
B-1	50.0		18		6	81	12			
B-2	5.0		10							
B-2	10.0		11		10	48	42			
B-2	20.0		8				21			
B-2	30.0		9							
B-2	40.0		20		0	82	18			

APPENDIX B

APPENDIX B

PREVIOUS EXPLORATIONS COMPLETED BY OTHERS

Test pit explorations completed in 1984 by Shannon & Wilson to support development of the existing building are presented in this appendix.

E. YESLER WAY

S. WASHINGTON ST.

16th AVE. S.

17th AVE. S.

TP-1

TP-2

TP-3

TP-4

TP-5

TP-6

TP-7

TP-8

APPROX. BUILDING LOCATION

EXIST. 8 UNIT APT., 2-STORY W/BSMT.

EXIST. HOUSE

EXIST. HOUSE

ASPHALT PARKING

ASPHALT PARKING

CONCRETE

LAND-SCAPED

WOOD FENCE ON CONC. WALL

6' CONC. WALL

WOOD FENCE

CHAIN-LINK FENCE

ROCKERY

10' CONC. WALL

CONC. WALK

CONC. CURB, TYP.

BUS STOP

GRASS

ASPHALT

CONC. WALL

CONC. WALK

CONC. WALK

CONC. WALK

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-1

SKETCH OF PIT SIDE SURFACE ELEVATION 243 feet

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose to medium dense, brown silty SAND with scattered wood and concrete fragments and gravel (Fill).	none observed	S-1	0
2. Dense, brown, silty, gravelly SAND (Glacial Till).			2
			4
			6
			8
			10
			12

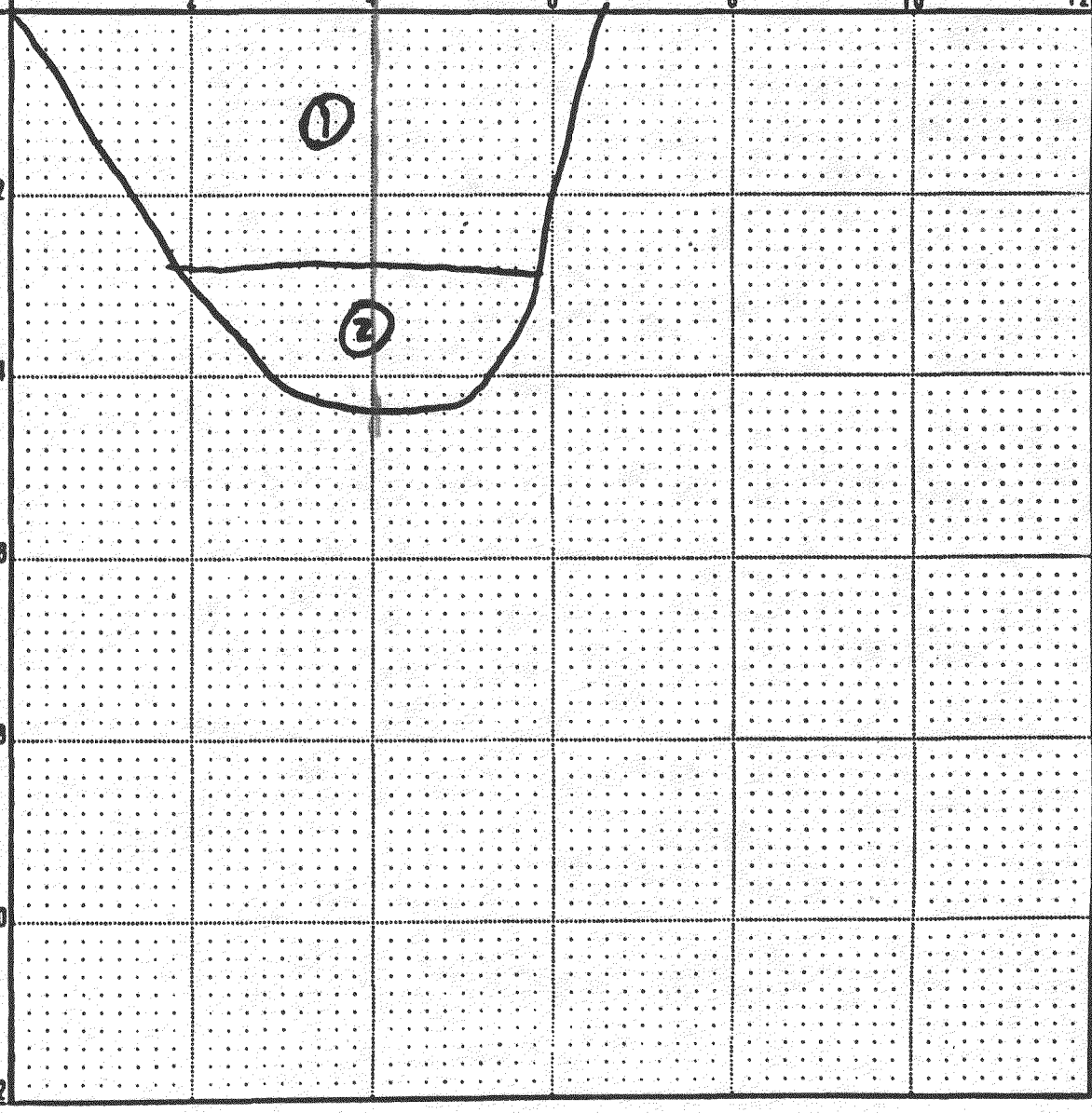


FIG. 2

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-2

SKETCH OF PIT SIDE SURFACE ELEVATION 239 feet

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, brown, silty, gravelly SAND with scattered organics, debris and wood fragments (Fill).	none observed	S-1	0
2. Dense, gray-brown, silty gravelly SAND (Glacial Till).			2
			4
			6
			8
			10
			12

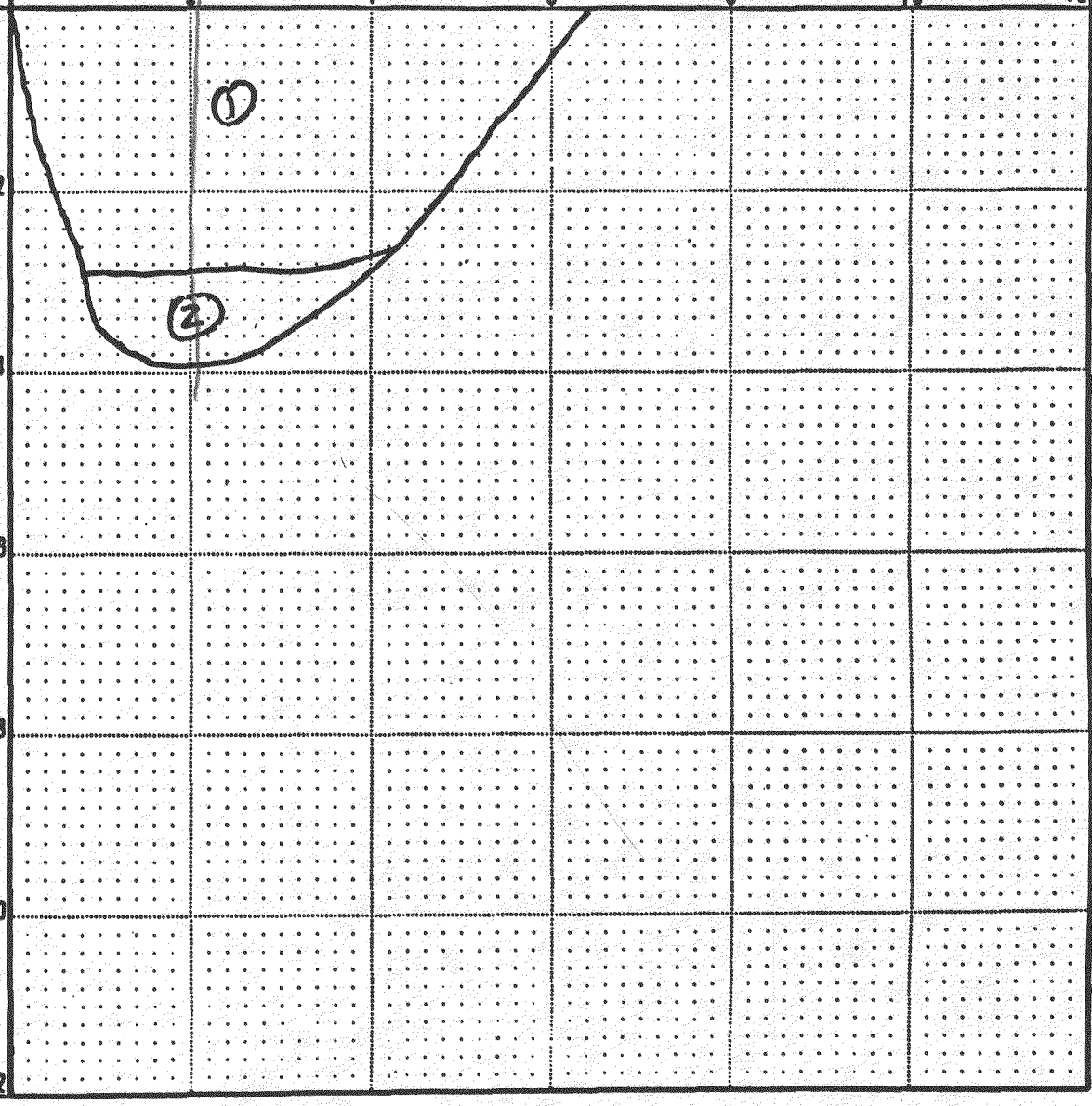


FIG. 3

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION see Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-3

SURFACE ELEVATION 247 feet

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, brown, gravelly silty SAND with concrete and asphalt fragments (Fill).	none observed	S-1	2
2. Medium dense, gray-brown, silty, gravelly SAND (Weathered Glacial Till).		S-2	4
3. Dense, gray, silty gravelly SAND (Glacial Till).			6
			8
			10
			12

SKETCH OF PIT SIDE

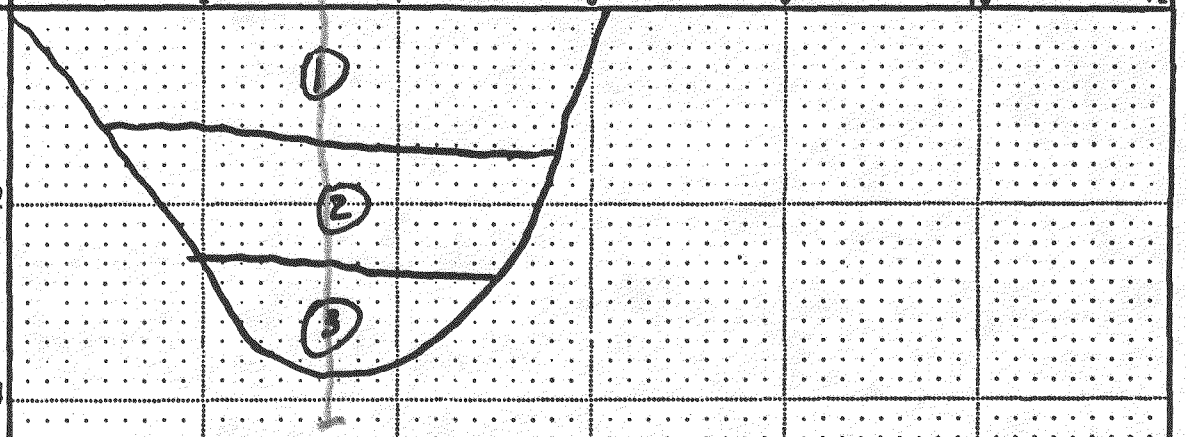


FIG. 4

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-4

SKETCH OF PIT SIDE SURFACE ELEVATION 248 feet

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, dark brown, silty, gravelly SAND. 2. Medium dense to dense, brown, silty, gravelly SAND.(Weathered Glacial Till). 3. Dense, gray, silty, gravelly SAND (Glacial Till).	none observed	S-1 S-2	0 2 4 6 8 10 12

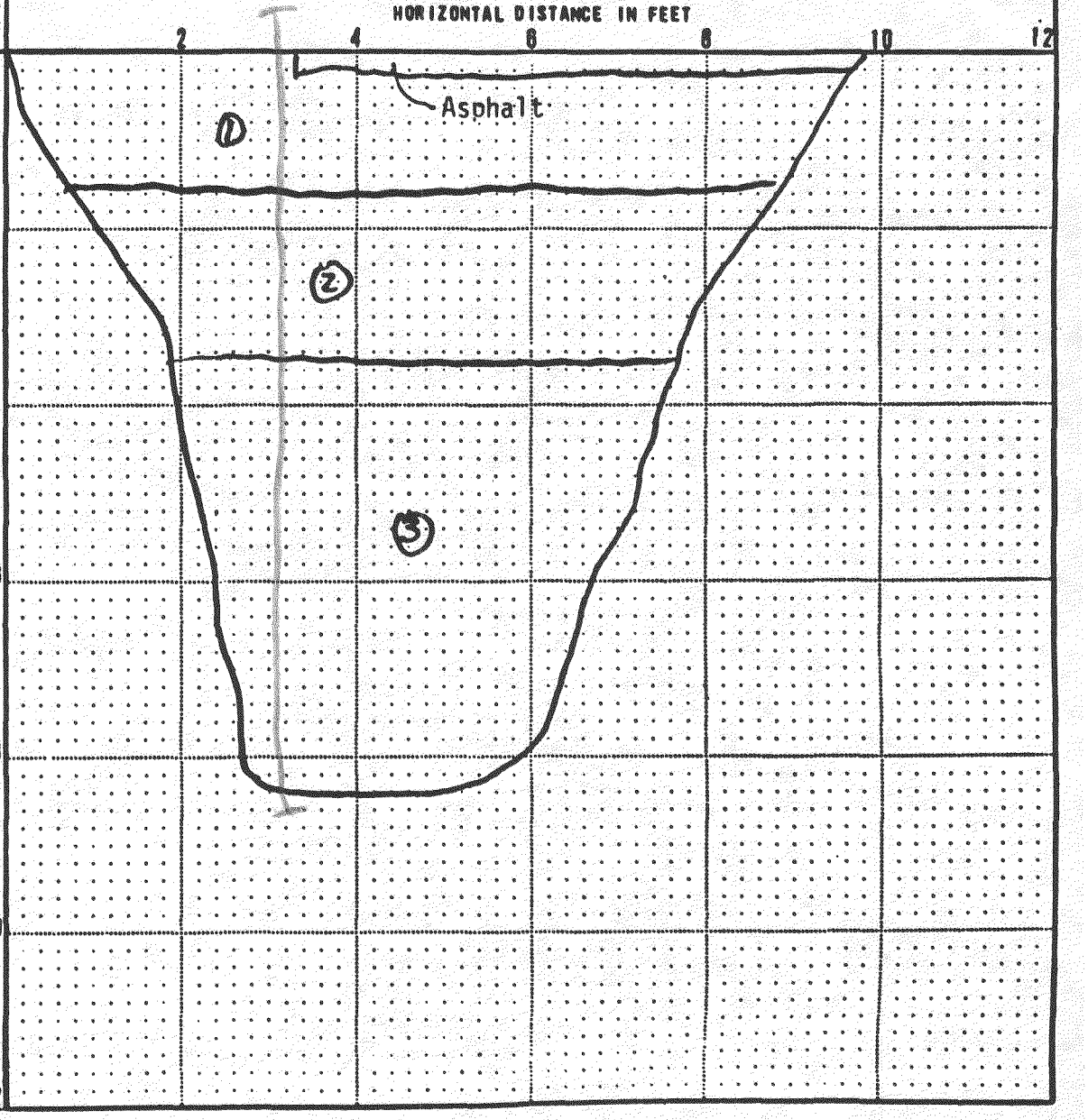


FIG. 5

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/1/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-5

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, black, silty, gravelly SAND with debris, roots and bottles (Fill).	none encountered	S-1	0
2. Loose, brown, silty, gravelly SAND. (Weathered Glacial Till).			2
3. Dense, gray, silty, gravelly SAND (Glacial Till).			4
			6
			8
			10
			12

SKETCH OF PIT SIDE SURFACE ELEVATION 254 feet

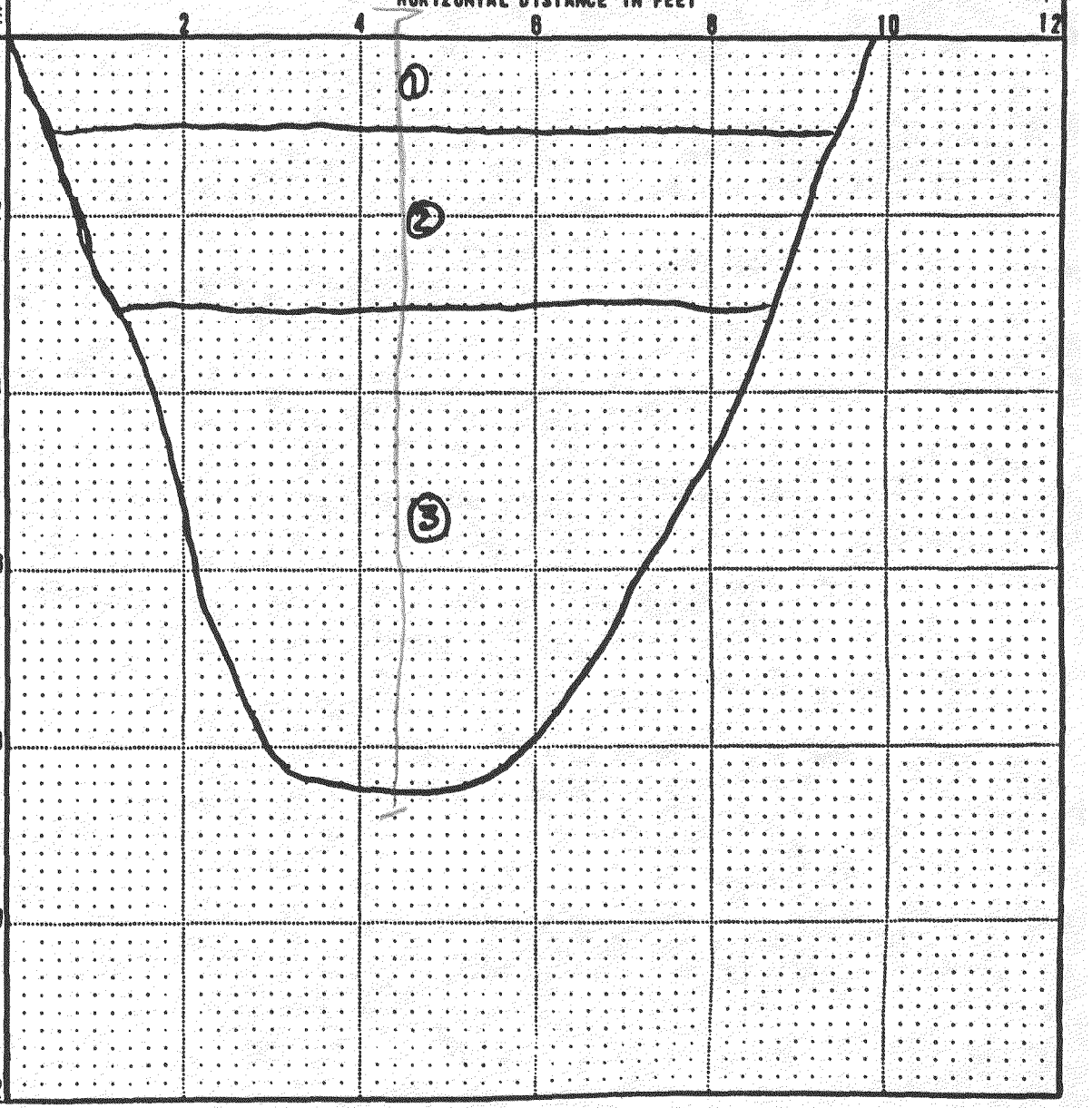


FIG. 6

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Figure 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-6

SKETCH OF N PIT SIDE SURFACE ELEVATION 246 feet

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, brown, silty, gravelly SAND with wood and concrete fragments, ashes (Fill). 2. Medium dense, brown, silty, gravelly SAND. (Weathered Glacial Till). 3. Dense, gray, silty, gravelly SAND (Glacial Till). Note: A wooden tie retaining wall with brick foundation was observed along the south edge of the test pit from 2.5 to 6.0 feet.	none observed	S-1	0 2 4 6 8 10 12

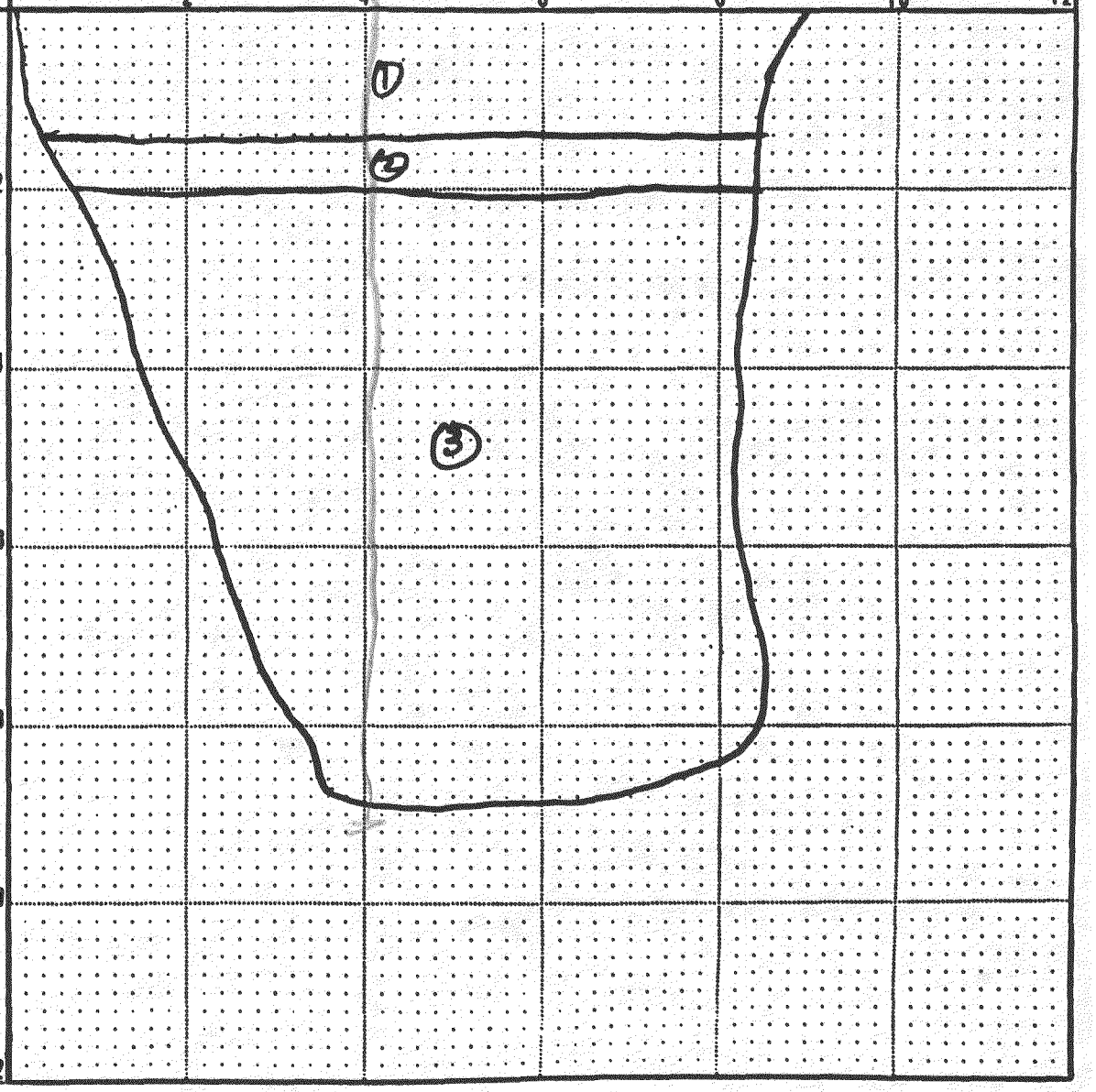


FIG. 7

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-7

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Loose, brown, silty gravelly SAND with scattered cobbles (Fill).	none observed	S-1	0
2. Medium dense, dark brown silty gravelly SAND with organic debris (Old Topsoil).			2
3. Medium dense, gray-brown, silty, gravelly SAND with scattered cobbles (Weathered Glacial Till).			4
4. Dense, gray, silty, gravelly SAND (Glacial Till).		S-2	8
			10
			12

SKETCH OF PIT SIDE SURFACE ELEVATION 253 feet

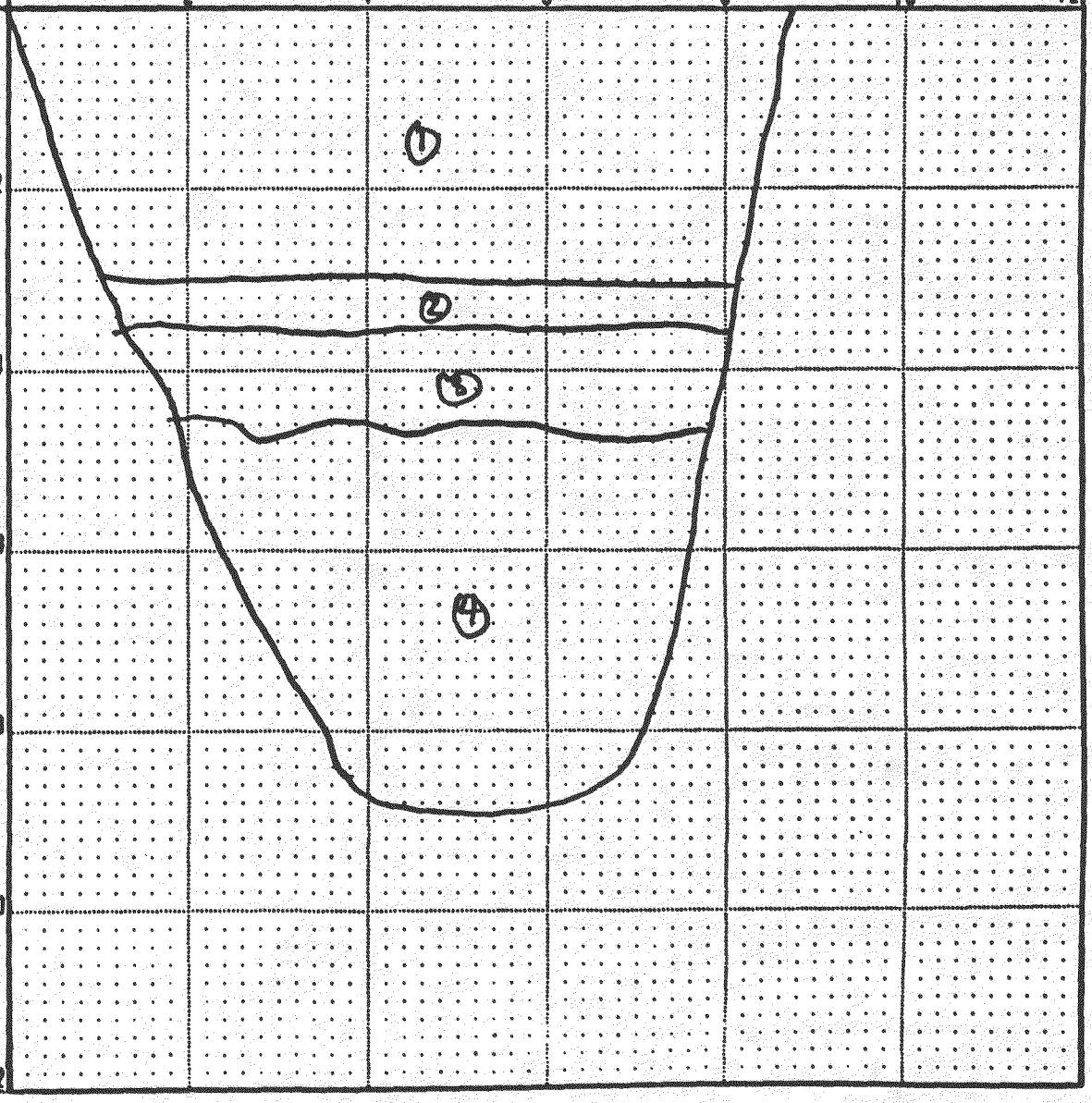


FIG. 8

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

JOB NO. W-4335-01 DATE 12/7/84 LOCATION See Fig. 1

PROJECT

INSPECTOR WEATHER

FIELD LOG OF TEST PIT TP-8

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES	DEPTH IN FEET
1. Medium dense, gray-brown, silty gravelly SAND. Weathered Glacial Till.	none encountered	S-1	2
2. Dense, gray, silty, gravelly SAND with scattered cobbles (Glacial Till).		S-2	4
			6
			8
			10
			12

SKETCH OF PIT SIDE SURFACE ELEVATION 251 feet

HORIZONTAL DISTANCE IN FEET

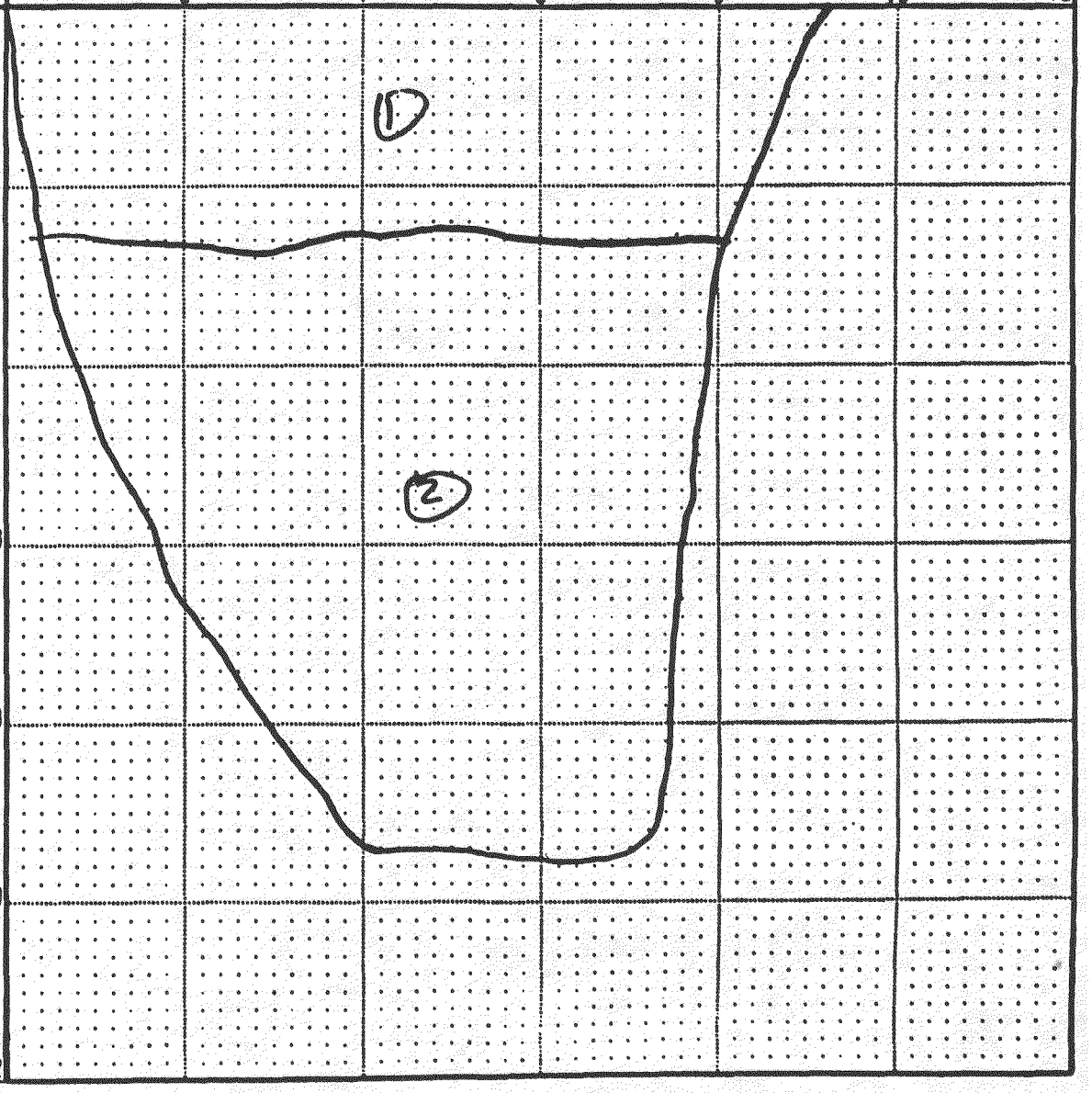
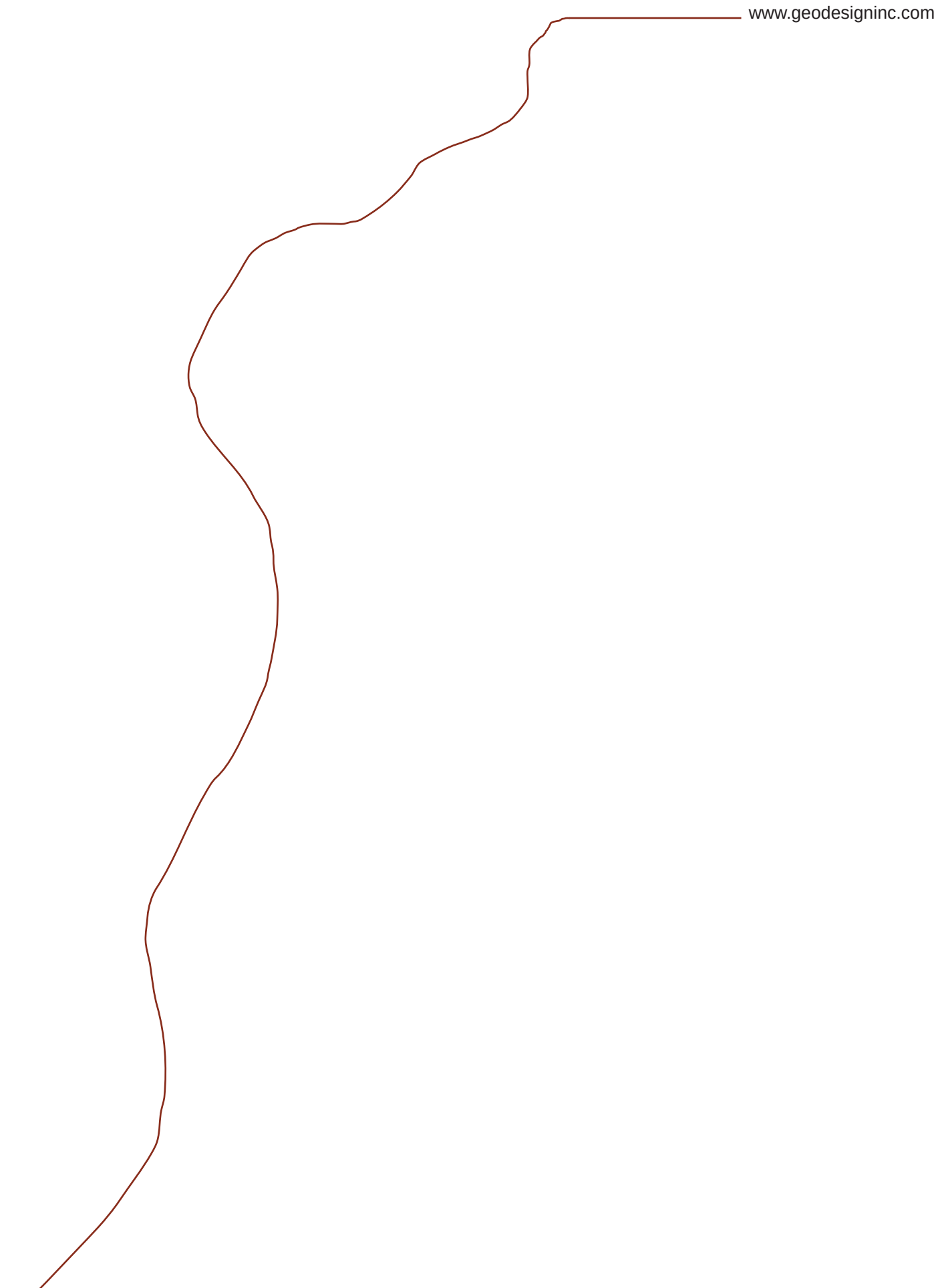


FIG. 9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